CHAPTER 15

STORAGE FACILITIES

Chapter 15 Storage Facilities
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15.1 Introduction

15.1.1 Overview

This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Storage and flood routing associated with culverts is addressed in the Culvert Chapter (note: criteria presented in this chapter do not necessarily apply to routine culvert design).

15.1.2 Detention And Retention

Stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those that are designed to detain runoff for some short period of time sufficient to reduce the peak discharge. They are designed using a dynamic stage-storage-discharge relationship. Retention facilities are designed to contain the flows until the storm has passed. They are drained by either gravity or pumping. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual. Ponds with a drainage basin less than 160 acres may be sized based on total retention of the inflow.

Storage facilities are also classified on the basis of whether they are dry or wet between storm events. A dry pond has on outlet positioned at or below the lowest elevation in the pond, such that the pond drains completely between storm events. A wet pond, however, has its lowest outlet at an elevation above the bottom of the pond. Water remains in the pond between storm events and is depleted only by infiltration or evaporation. Only dry ponds will be discussed further.

Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

15.1.3 Regional versus On-Site Detention

Stormwater storage facilities may be designed to handle drainage that is local to the highway system, address off-site flows coming to the highway or be part of a larger regional system. The type of basin and cooperating stakeholders will affect the design criteria and methods.

15.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 15-1 Symbols And Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
C	Weir coefficient	—
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
f	Infiltration rate	ft/hr
g	Acceleration due to gravity	ft/sec^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Infiltration rate	mm/hr
I	Inflow rate	ft^3/sec
L	Length	ft
O	Outflow rate	ft^3/sec
Q	Flow	ft^3/sec
S,Vs	Storage volume	ft^3, Ac-ft
t	Routing time period	sec
t_b	Time base on hydrograph	hrs
T_i	Duration of basin inflow	hrs
t_p	Time to peak	hrs
W	Width of basin	ft
z	Side slope factor	—

ADWR is the Arizona Department of Water Resources. This agency has jurisdiction over dams.

Emergency spillway is the outlet location where water will flow when it exceeds the storage capacity of the basin. The location and orientation shall be selected with consideration of the impact of the direction that water will outflow.

Jurisdictional Dam is a structure subject to ADWR jurisdiction, structures that store less than 50 Acre-feet and have an impoundment height of less than 25 feet are not considered jurisdictional. See Appendix A.

Principal spillway is the outlet structure through which it is intended that the design outflow occur without requiring flow through the emergency outlet.

Freeboard is the vertical distance from the maximum water surface to the lowest bank elevation.

15.3 Design Objectives And Concepts

15.3.1 Introduction

Stormwater storage basins are used to reduce the impact of stormwater flows on downstream areas. The impact concerns may be of quantity and/or quality. The most common desired outcome is a lower peak flow. The process involves the directing of stormwater runoff into a basin and controlling the outflow to a controlled lower rate though a properly sized outlet. The outflow hydrograph will have a lower peak discharge and a longer duration of discharge.

15.3.2 Quantity Goals

Quantity goals for storage facilities may be based on:

- prevention or reduction of peak runoff rate increases caused by changes in the watershed,
- mitigation of downstream drainage capacity problems,
- reduction or elimination of the need for downstream outfall improvements, and
- recharging of groundwater resources.

The objectives for stormwater quantity are typically based on limiting the peak discharge rates to match one or more of the following values:

- historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for specified frequency of occurrence.)
- limiting risks based on the hydraulic capacity of the downstream drainage system, or
- a specified value set by jurisdictional regulation or agreement (such as determined by joint project agreements for downstream improvements).

Location Considerations

The utility of providing a storage facility depends on the amount of storage, its location within the system and its operational characteristics. In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. Thus it is important for the engineer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the watershed. Multiple storage facilities located in the same watershed will affect the timing of the runoff through the conveyance system that could result in a decrease or increase of flood peaks in different downstream locations. If several storage facilities are located within a particular watershed it is important to determine what effects a particular facility may have on combined hydrographs in down-stream locations. Effective stormwater management must be coordinated on a regional or basin-wide basis.

The evaluation of storage facilities should include comparison of the design flow at a point or points downstream of the proposed storage site with and without the additional storage. This may require channel routing calculations to be carried downstream to a point where the effect of the proposed storage facility hydrograph on the downstream hydrograph can be assessed for detrimental impacts on downstream areas.

For some installations, it may be feasible and desirable to by-pass some portion of the approach flow. This is accommodated for in the design by adjustment of the peak outflow and by-pass flow to meet the desired downstream discharge. This is especially useful if the by-pass flow is high enough so that very common storm events are not diverted into the storage basin.

15.3 Design Concepts & Objectives (continued)

15.3.3 Quality Goals

Quantity goals for storage facilities may be based on:

- control of sediment deposition and
- providing filtration or capture of first flush pollutants.

15.3.4 Basin Design Concept

The basic concept associated with detention basins is the attenuation of flow through the application of a routing analysis for a given pond, outlet configuration and inflow hydrograph. At any given time interval one of the following possibilities is occurring:

- If the average inflow rate is larger than the average outflow rate for the given time interval, the volume of water stored increased, the water surface increased, and the average outflow rate for the next time interval increased.
- If the average inflow rate is equal to the average outflow rate for the given time interval, the volume of storage and the water surface stayed constant.
- If the average inflow rate is less than the average outflow rate for the give time interval, the volume of water stored decreased, the water surface decreased, and the average outflow rate for the next time interval decreased.

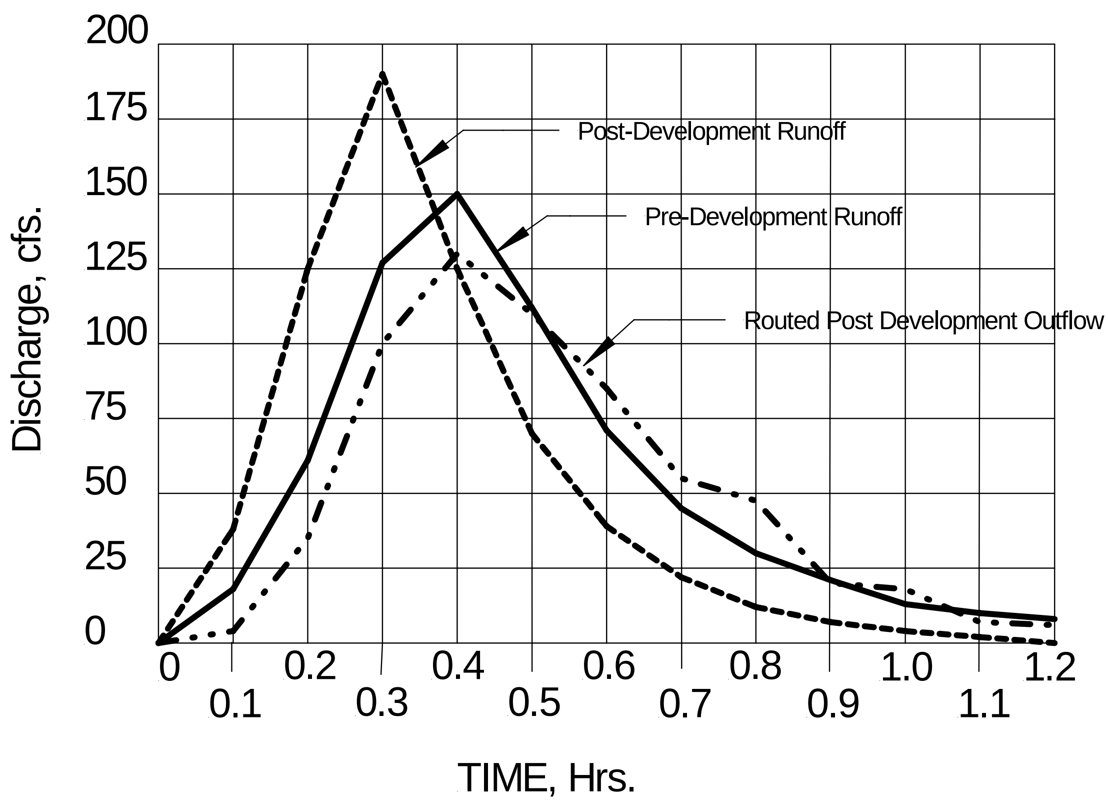

Fig. 15.1 Detention Routing: Inflow-Outflow hydrograph

15.3 Design Concepts & Objectives (continued)

15.3.4 Basin Design Concept (continued)

The volume under the inflow hydrograph is the total volume of runoff entering the basin. The volume under the outflow hydrograph is the total volume of runoff leaving the basin. The change in the value of the peak discharge and the time of peak flow is accomplished by the attenuation of the flow through the basin. The volume above the outflow hydrograph and below the inflow hydrograph is the amount of storage required. The maximum volume occurs when the two hydrographs intersect, which is also when the maximum outflow rate occurs.

For most detention basins the outflow outlet is a culvert or weir structure that is uncontrolled. For a basin with an uncontrolled outlet to a free outfall, the peak storage and the peak outflow will occur at the point where the outflow hydrograph intersects the inflow hydrograph.

A general procedure using the above data in the design of storage facilities is presented below. This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

Step 1 Compute inflow hydrograph for runoff from the appropriate design storms using the procedures outlined in the ADOT Hydrology Manual. Both pre- and post-development hydrographs may be required.

Step 2 Perform preliminary calculations to estimate detention storage requirements for the hydrographs from Step 1 (see Section 15.7). When looking at a range of storms, if storage requirements are satisfied for runoff from the minimum and maximum design storm events, runoff from intermediate storms is assumed to be controlled. The maximum storage requirement calculated should be used.

Step 3 Determine the physical dimensions necessary to hold the estimated volume, including freeboard.

Step 4 Size the outlet structure. The estimated peak stage will occur for the estimated maximum volume. The outlet structure should be sized to convey the allowable discharge at this peak stage.

Step 5 Perform routing calculations using inflow hydrographs to check the design using the storage routing equations. If the routed outflow peak discharges exceed the desired outflow peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated basin volume and return to step 3.

Step 6 When a satisfactory initial basin size is determined, consider emergency overflow from runoff due to storms larger than the design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being retained represents 10% of the total drainage area.

Step 8 Evaluate the structure outlet velocity and provide channel and bank stabilization if the velocity would cause erosion problems downstream.

15.4 Design Criteria

15.4.1 General Criteria

In addition to the operational design flow, flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis (i.e., 100-year flood), especially for the spillway. The design criteria for storage facilities should include:

- release rate,
- grading and depth requirements,
- outlet connection and location,

These will be used to determine the
- outlet size,
- storage volume, and
- shape/layout of basin.

15.4.2 Release Rate

Release rates shall approximate the desired/required peak runoff rates for the design storm events, with the emergency overflow capable of handling flows in excess of the design storms. In addition, the release rate shall accomplish draining of all detention volume within 36 hours after the cessation of the storm event.

The storage facility may be required to meet release rates for a range of design storms. This may be accomplished using multi-stage control structures (Figure 15-12). If a multi-stage control structure satisfies the upper and lower discharge requirements, discharges from intermediate storm return periods can usually be assumed to be adequately controlled.

The emergency spillway should be capable of passing flows up to 120% of the 100-year storm.

15.4.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to the values required as stated above. To achieve these rates, the basin's storage should be equal to the area between the pre- and post-construction hydrographs.

15.4.4 Grading And Depth

15.4.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Storage facilities situated above ground may become subject to the Arizona Revised Statue (A.R.S.) 45-1201 Dams and Reservoirs as administered by the Arizona Department of Water Resources (ADWR). Structures that have impoundment depths greater than 6 ft and storage volumes greater than 15 Ac-Ft may be come under the jurisdiction of the ADWR unless the facility is excavated below existing ground. See Appendix A for ADWR jurisdiction criteria.

15.4 Design Criteria (continued)

15.4.4.1 General (continued)

It is ADOT's goal not to construct or own any facilities that come under ADWR jurisdiction. ADOT prefers to create storage facilities by excavation below existing ground.

A basin can have multiple levels: one to hold the smaller storms, and a second, which is rarely inundated and can be used for other purposes, to store the larger storms. Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urban areas.

A minimum freeboard of 1 foot above the design storm high water elevation shall be provided.

Storage facilities shall be designed to address the following maintenance concerns:

- maintenance of fences and perimeter plantings.
- grass and vegetation maintenance,
- bank deterioration,
- blockage of outlet structures,
- litter accumulation
- weed growth,
- standing water or soggy surfaces,
- sedimentation control,
- mosquito control,

The following sections provide guidelines that should minimize maintenance problems.

15.4.4.2 Grading

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff.

Side slopes are limited to those that are compatible with the landscape treatment; 2.5:1 for desert/rock slopes and 3:1 for grass slopes.

The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions. If sediment deposition is expected, an initial sedimentation trap basin with access should be considered. Access ramps shall be provided for maintenance equipment. Generally, 25 feet is provided between the top of the basin and the right-of-way line.

15.4 Design Criteria (continued)

15.4.5 Inlet and Outlet Works

Inlet structures may be necessary where the inflow is being dropped into the basin. Riprap or other energy dissipation measures may be necessary. If by-pass flows are desired, then a diversion structure is necessary in the upstream channel. Diversion structures are of two types. One type diverts a fixed percentage of the approach flow. This type of diversion is accomplished using a flow splitter. The second type diverts the discharge above a base flow. Flows below the base level are not captured into the detention basin. Diversion of flows above a base level is more effective at reducing the required size of storage

The outlet structure allows flows to discharge from the storage basin at a controlled rate. Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow. The principal spillway is intended to convey the design storm without requiring flow to enter an emergency outlet. Outlets can be designed in a wide variety of configurations. Outlet works may be sized to vary the outflow with the varying depth. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices at various levels. Slotted riser pipes are discouraged because of clogging problems. If the outlet is a pipe through an embankment, then an anti-seepage collar should be provided to minimize piping by water leakage of the soil particles surrounding the pipe.

For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 120% of the 100-year flood. The emergency outlet shall be designed to operate under extreme conditions to prevent failure of the retention structure. The outlet may need to be protected with riprap or paving to prevent excessive damage to the spillway if it would be subjected to high velocity flows. Design information regarding spillway hydraulics is available in *Design of Small Dams*, USBR, 1987.

15.4.6 Access/Protective Treatment

Protective treatment may be required to prevent entry to facilities that present a hazard. Safety considerations include fencing the basin, reducing the maximum depth and/or including ledges and mild slopes to prevent people from falling in and facilitate their escape from the basin. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible.
- Water depths exceed 3 feet for more than 24 hours.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 0.5 ft or a flow velocity greater than 5 ft/s.
- Side slopes equal or steeper than 2:1.

15.4 Design Criteria (continued)

15.4.6 Access/Protective Treatment (continued)

ADOT concurrence shall be obtained when the basin is not to be fenced. Grates or fencing may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

15.5 General Procedure

15.5.1 Basic Concept

The basic concept involved in storm water detention analysis is a routing procedure using the storage-indication method to transform the inflow hydrograph into the outflow hydrograph. Routing calculations needed to analyze storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs.

The routing calculations are based on the Puls method. To perform the calculations one must have the following data:

- Inflow hydrograph for all selected design storms. Generally derived by use of HEC-1.
- Stage-storage curve for proposed storage facility (see section 15.5.2). For large storage volumes, use Acre-Feet, otherwise use cubic feet.
- Stage-discharge curve for all outlet control structures (see Section 15.5.3).

Using these data, a routing procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see Section 15.8). The computation begins with inflow. At a given time interval, the inflow is known. Using the storage at the end of the previous time interval, add the inflow volume to get the intra-period Stage, using the intra-period stage get the outflow rate. Use the outflow rate to get the intra-period outflow volume. Use the beginning of period volume plus the intra-period inflow volume minus the intra-period outflow volume to determine the end of period volume. The next time interval is then calculated.

The general procedure described above is presented below. See Section 15.8 for a detailed description of each step.

Step 1 Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3 Use the storage-discharge and stage-storage data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 15-4.

15.5 General Procedure (continued)

15.5.1 Basic Concept (continued)

Step 4 I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, S_1-$(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 15-10).

Step 5 Determine the value of $S_2 + (O_2/2)\ \Delta t$ from the following equation

$$S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2]\Delta t \qquad (15.2)$$

Where: S_2 = storage volume at time 2, ft^3
O_2 = outflow rate at time 2, ft^3/sec
Δt = routing time period, sec
S_1 = storage volume at time 1,ft^3
O_1 = outflow rate at time 1, ft^3/sec
I_1 = inflow rate at time 1, ft^3/sec
I_2 = inflow rate at time 2, ft^3/sec
Other consistent units are equally appropriate.

Step 6 Enter the storage characteristics curve at the calculated value of S_2+$(O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7 Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8 Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

15.5.2 Stage-Storage Curve

A stage-storage curve defines the incremental relationship between the depth of water and storage volume in a reservoir, Figure 15-3. Storage basins may be irregular in shape to blend well with the surrounding terrain. The data for this type of curve are usually developed using a topographic map and one of the following formulas: the average-end area, frustum of a pyramid, or prismoidal formulas. The average-end area formula is usually preferred as the method to be used on non-geometric areas. This is usually developed from the stage versus surface area data. The information should extend from the basin invert to the top of the embankment. The precision of the information increases as the interval of stage decreases.

15.5 General Procedure (continued)

15.5.2 Stage-Storage Curve (continued)

The **double-end area** formula is expressed as:

$$\mathbf{V_{1,2} = [(A_1 + A_2)/2]d} \qquad \textbf{(15.3)}$$

Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2
d = change in elevation between points 1 and 2, ft

The **frustum of a pyramid** is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2]} \qquad \textbf{(15.4)}$$

Where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2

The **prismoidal formula** for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + (4/3)\ Z^2\ D^3} \qquad \textbf{(15.5)}$$

Where: V = volume of trapezoidal basin, ft^3
L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of vertical to horizontal

15.5.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the stage (depth) of water and the discharge or outflow from a storage facility, Figure 15-3. This curve is also called a *rating curve*. A typical storage facility has two spillways: principal and emergency. The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

Tailwater influences and structure losses must be considered when developing discharge curves. If a combination of outlet structures is used, backwater effects of one structure may affect the discharge of the combination of structures. Section 15.6 presents methods for calculating the stage-discharge relationship of weirs. The culvert chapter presents methods for calculating the stage-discharge relationship for structures having with a Hw/D ratio greater than 1.2 or affected by tailwater. The precision of the information increases as the interval of stage decreases.

15.5 General Procedure (continued)

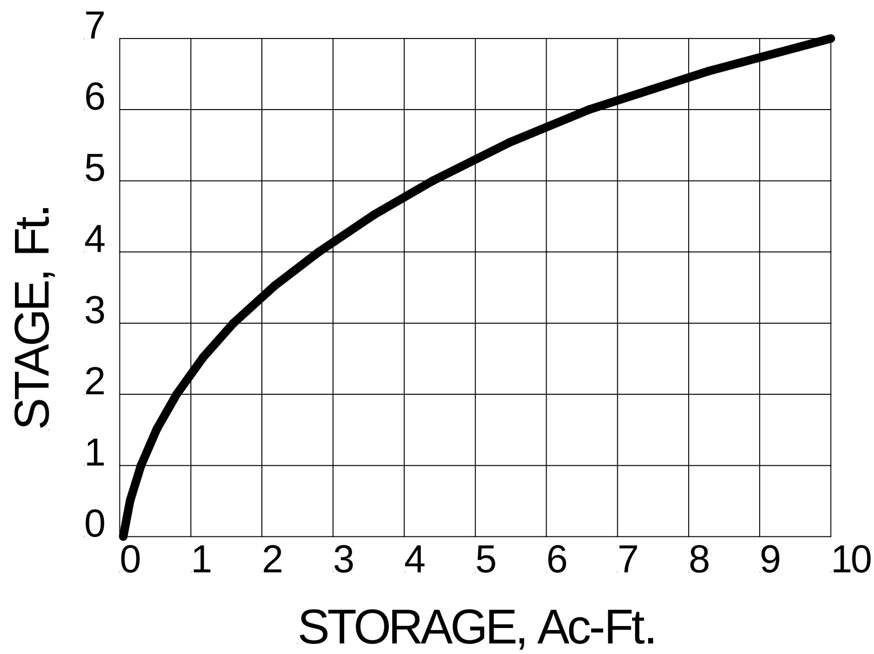

Figure 15-2 Example Stage-Storage Curve

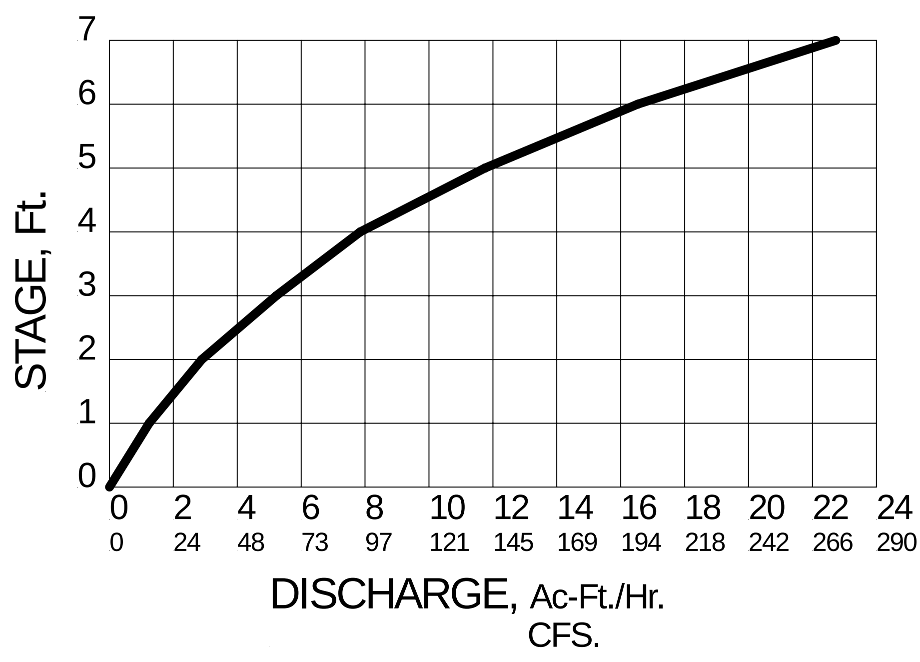

Figure 15-3 Example Stage-Discharge Curve

15.6 Outlet Hydraulics

15.6.1 General

Sharp-crested weir flow equations for no end contractions, two end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data. When analyzing release rates the tailwater influence on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening.

15.6.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 15-4. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_c)]\ LH^{1.5}} \qquad \mathbf{(15.6)}$$

Where: Q = discharge, ft^3/sec
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

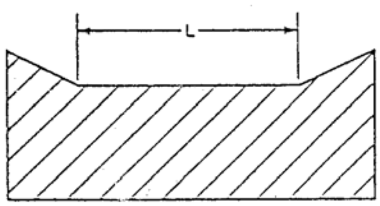

Figure 15-4

Sharp-Crested Weir (No End Contractions)

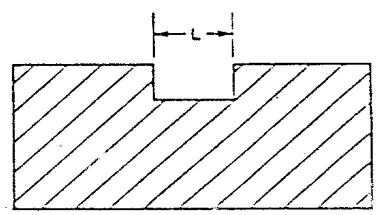

Figure 15-5

Sharp-Crested Weir (Two End Contractions)

A sharp-crested weir with two end contractions is illustrated in Figure 15-5. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_c)]\ (L - 0.2H)\ H^{1.5}} \qquad \mathbf{(15.7)}$$

Where: Variables are the same as equation 15.6.

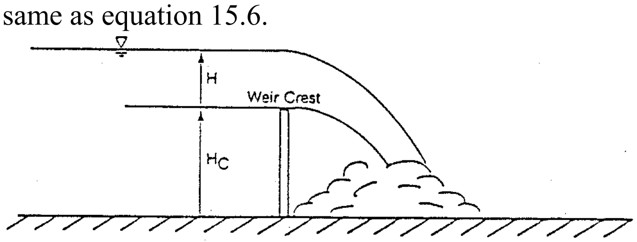

Figure 15-6 Sharp-Crested Weir And Head

15.6 Outlet Hydraulics (continued)

15.6.2 Sharp-Crested Weirs (continued)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \tag{15.8}$$

Where: Q_s = submergence flow, ft^3/sec
Q_f = free flow, ft^3/sec
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

15.6.3 Broad-Crested Weirs

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \tag{15.7}$$

Where: Q = discharge, ft^3/sec
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 15-3.

15.6 Outlet Hydraulics (continued)

15.6.3 Broad-Crested Weirs (continued)

Table 15-3 Broad-Crested Weir Coefficient C Values
As A Function Of Weir Crest Breadth And Head (ft)

Measured Head, H[1] (Ft)	Breadth Of The Crest Of Weir (Ft) 0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.0	15.0
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.22	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.32	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

15.6 Outlet Hydraulics (continued)

15.6.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$\mathbf{Q = 2.5\ tan(\theta/2)H^{2.5}} \qquad \mathbf{(15.10)}$$

Where: Q = discharge, ft^3/sec
θ = angle of v-notch, degrees
H = head on apex of notch, ft

15.6.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are (Sandvik, 1985):

$$\mathbf{Q = 4.97\ a^{0.5}\ b(H - a/3)} \qquad \mathbf{(15.11)}$$
$$\mathbf{x/b = 1 - (1/3.17)\ (arctan\ (y/a)^{0.5})} \qquad \mathbf{(15.12)}$$

Where: Q = discharge, ft/sec
Dimensions a, b, H, x and y are shown in Figure 15-7.

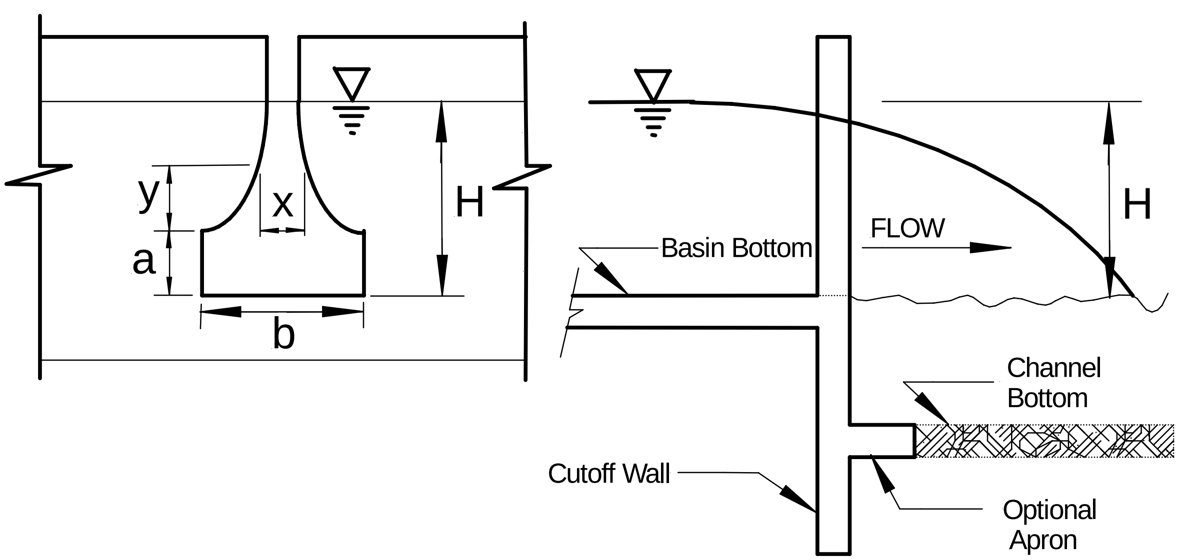

Figure 15-7 Proportional Weir Dimensions

15.6 Outlet Hydraulics (continued)

15.6.6 Orifices

Pipes smaller than 12 inch may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

$$\mathbf{Q = 0.6A(2gH)^{0.5} = 3.78\ D^2H^{0.5}} \qquad \mathbf{(15.13)}$$

Where: Q = discharge, ft^3/sec
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/sec^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface, ft *

- In cases where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.

15.7 Preliminary Detention Volume Calculations

15.7.1 Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 15-8 shown below.

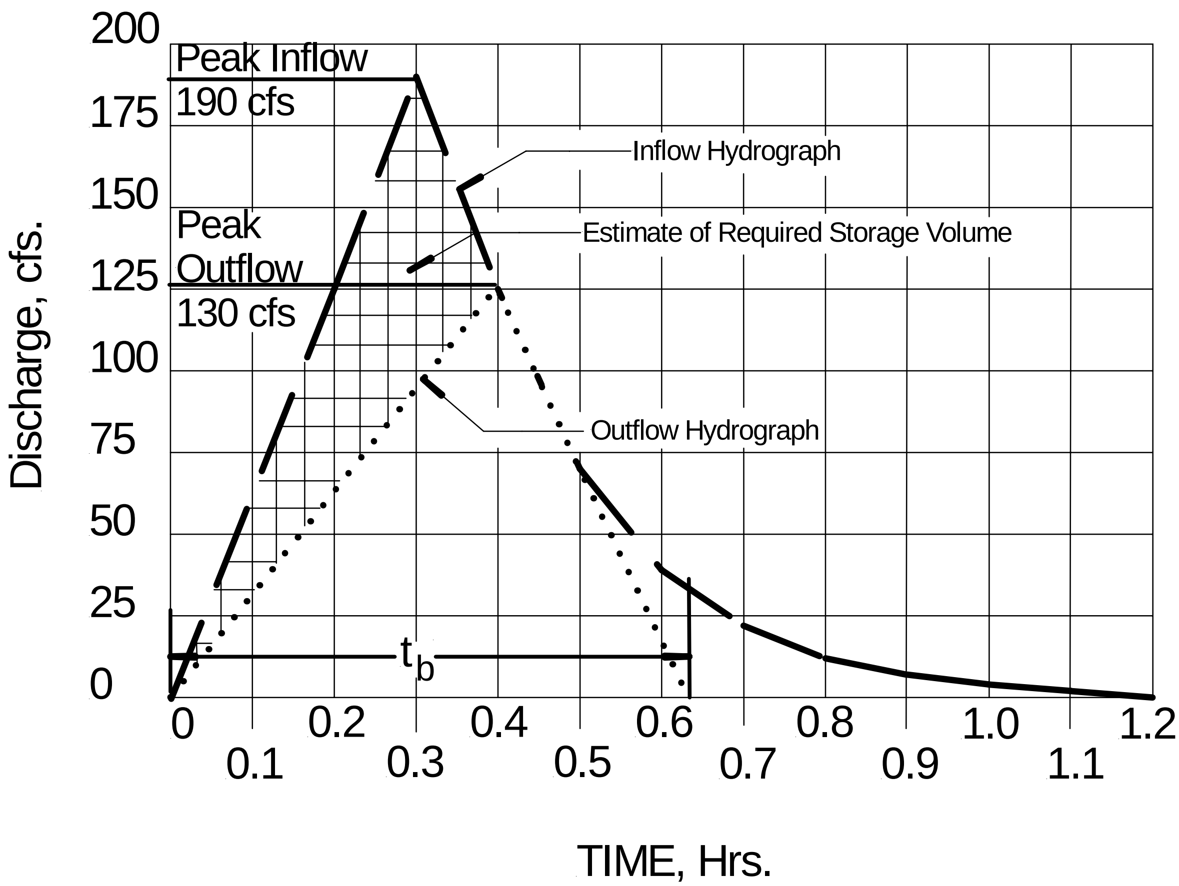

Figure 15-8 Triangular Shaped Hydrographs

(For Preliminary Estimate Of Required Storage Volume)

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_b(Q_i - Q_o) \qquad (15.14)$$

Where: V_S = storage volume estimate, ft^3
Q_i = peak inflow rate, ft^3/sec
Q_o = peak outflow rate, ft^3/sec
T_b = duration of basin inflow, t_b

Any consistent units may be used for Equation 15.14.

15.7 Preliminary Detention Volume Calculations (continued)

15.7.1 Storage Volume (continued)

For basins which have a contributing drainage area less than 160 acres, the storage volume may be estimated assuming that all flow is retained. The volume is estimated as

$$V=CAP_{24}$$

Where: V = storage volume estimate, Ac-ft
C = Rational Runoff coefficient
A = Contributing drainage area, Acre
P_{24} = 24 hour rainfall amount, ft.

15.8 Routing Calculations

The routing calculations are based on the Puls method. The fundamental principle is the conservation of mass, i.e, the change in storage during a time interval is equal to the difference between the inflow and outflow volumes. The method described below is a finite difference solution of the conservation of mass relationship, it is commonly known as the storage-indication method. The calculations are the process of analyzing the difference between the flow entering and the flow leaving the basin for a series of time increments. The difference determines the change in volume and water surface elevation.

To perform the calculations one must have the inflow hydrograph, the stage-storage and stage-discharge relationships. The computation begins with inflow. At a given time interval, the inflow is known. Using the storage at the end of the previous time interval, add the inflow volume to get the intra-period Stage, using the intra-period stage get the outflow rate. Use the outflow rate to get the intra-period outflow volume. Use the beginning of period volume plus the intra-period inflow volume minus the intra-period outflow volume to determine the end of period volume. The next time interval is then calculated.

The general procedure described above is presented below.

Step 1 Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. Example stage-storage curve is shown below in Figure 15-9, and stage-discharge curve is shown in Figure 15-10.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3 Use the storage-discharge and stage-storage data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 15-4.

Step 4 I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1-(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 15-10).

15.8 Routing Calculations (continued)

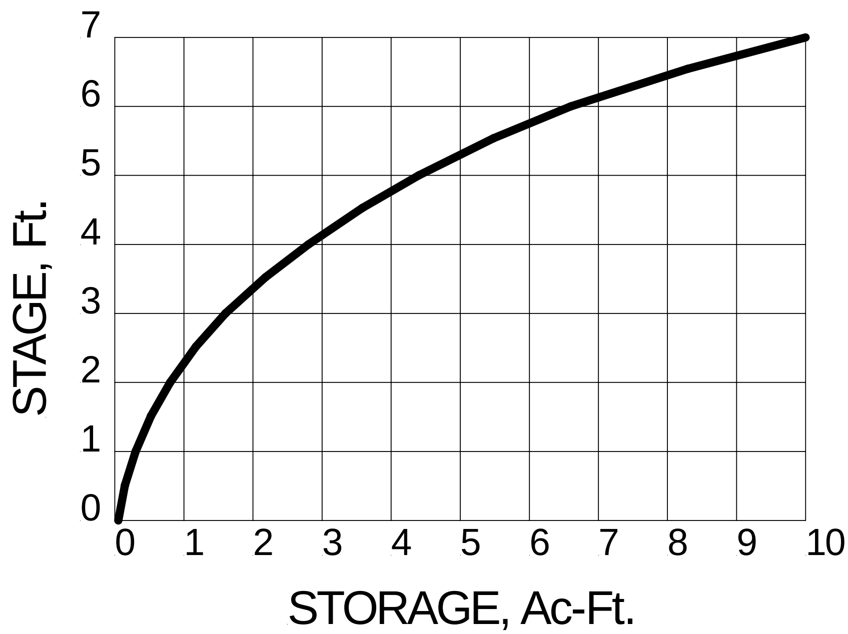

Figure 15-9 Example Stage-Storage Curve

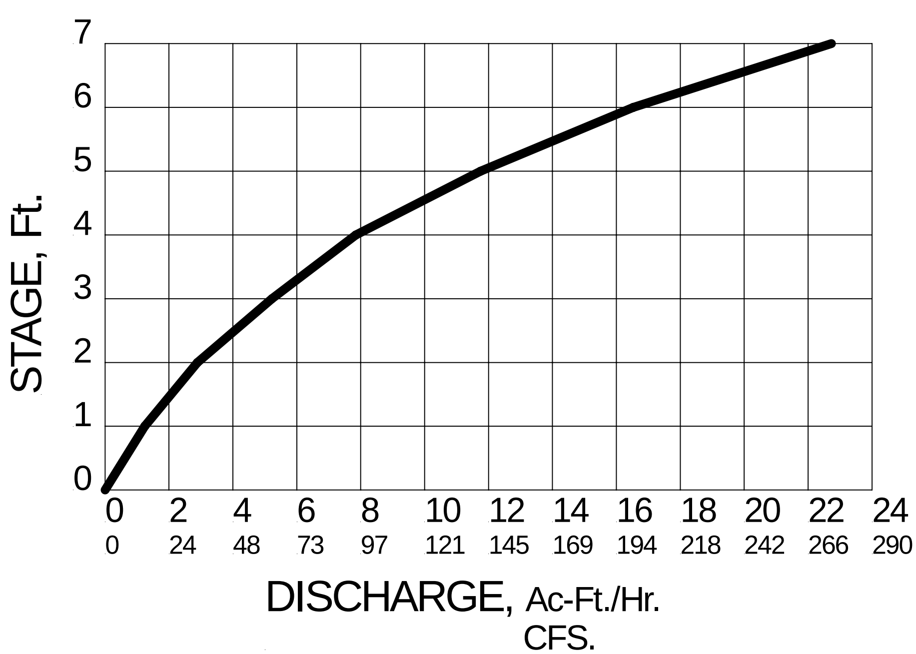

Figure 15-10 Example Stage-Discharge Curve

15.8 Routing Calculations (continued)

Table 15-4 Storage Characteristics

(1) Stage (ft)	(2) Storage[1] (Ac-Ft)	(3) Discharge[2] (ft^3/s)	(4) (Ac-Ft/hr)	(5) S-(O/2)Δt (Ac-Ft)	(6) S+(O/2)Δt (Ac-Ft)
100	0.05	0	0.0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.72	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[1] Obtained from the Stage-Storage Curve.
[2] Obtained from the Stage-Discharge Curve.
Note: Δt = 10 min = 0.167 h, and 1cfs=0.0826 ac-ft/hr.

Step 5 Determine the value of $S_2 + (O_2/2)\,\Delta t$ from the following equation

$$\mathbf{S_2 + (O_2/2)\,\Delta t = [S_1 - (O_1/2)\,\Delta t] + [(I_1 + I_2)/2]\Delta t} \qquad \mathbf{(15.15)}$$

Where: S_2= storage volume at time 2, ft^3
O_2 = outflow rate at time 2, ft^3/sec
Δt = routing time period, sec
S_1 = storage volume at time 1,ft^3
O_1 = outflow rate at time 1, ft^3/sec
I_1 = inflow rate at time 1, ft^3/sec
I_2 = inflow rate at time 2, ft^3/sec
Other consistent units are equally appropriate.

Step 6 Enter the storage characteristics curve at the calculated value of $S_2+(O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7 Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8 Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

15.8 Routing Calculations (continued)

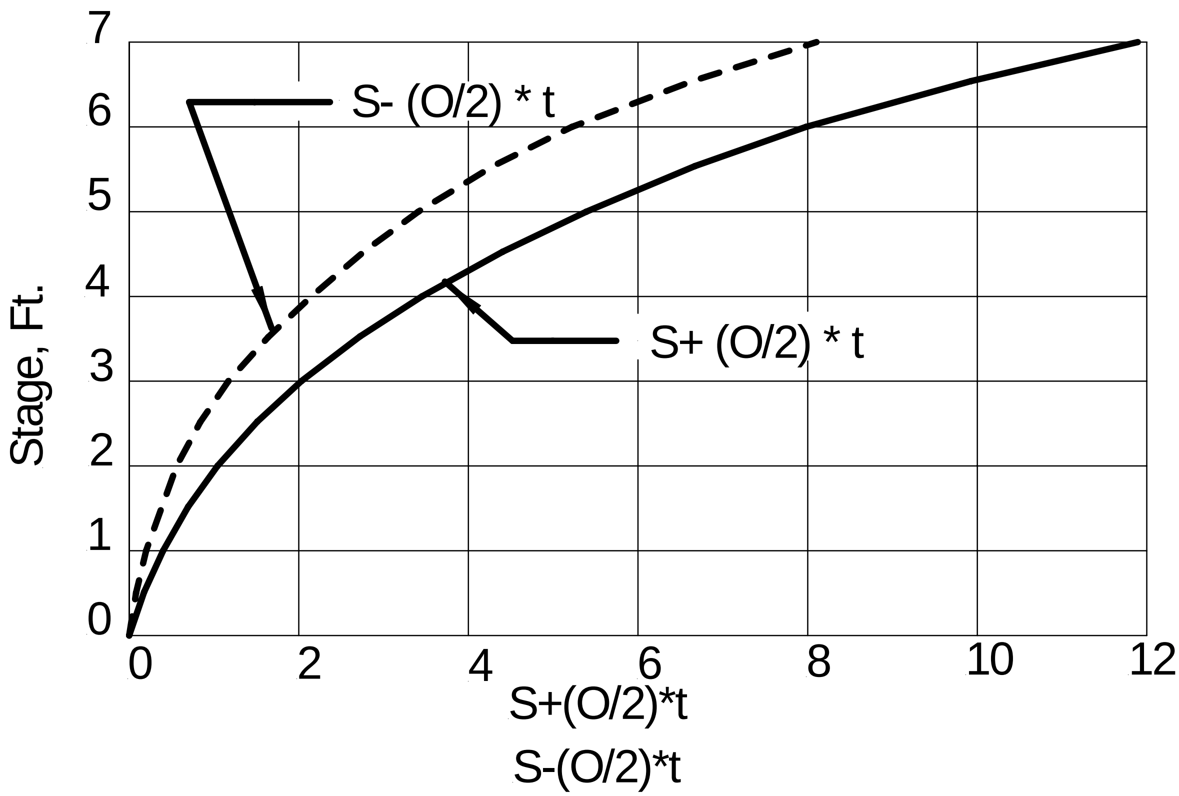

Figure 15-11 Storage Characteristic Curve

15.9 Example Problem

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions have been developed.

15.9.1 Design Discharge And Hydrographs

Storage facilities are to be designed for runoff from both the 2- and 10-year design storms and an analysis done using the 100-year design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Runoff hydrographs are shown in Table 15-5 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 h, respectively, for runoff from the 2- and 10-year storms. Peak discharges from the 2- and 10-year design storm events are as follows:

Pre-development

2-year peak discharge	10-year peak discharge
150 ft^3/s	200 ft^3/s

Post-development

2-year peak discharge	10-year peak discharge
190 ft^3/s	250 ft^3/s

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 ft^3/s for the 2- and 10-year storms, respectively.

Table 15-5

Runoff Hydrographs

	Pre-Development Runoff		Unrouted Post-Development Runoff	
(1) Time (h)	(2) 2-Year (ft^3/s)	(3) 10-Year (ft^3/s)	(4) 2-Year (ft^3/s)	(5) 10-Year (ft^3/s)
0.0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190	250
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

15.9 Example Problem (continued)

15.9.2 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 15.8. For runoff from the 2- and 10-year storms, the required storage volumes, V_S, are computed using equation 15.13:

$V_S = 0.5T_i(Q_i - Q_o)$
Using a time base of 0.62 hours (figure 15-8) for the 2-year event and 0.7 hours for the 10-year event,

2-year storm: $V_S = 0.5(0.62 \times 3600)(190\text{-}150)/43560 = 1.28$ Ac-Ft
10-year storm: $V_S = 0.5(0.70 \times 3600)(250\text{-}200)/43560 = 1.45$ Ac-Ft

Table 15-6

Stage-Discharge-Storage Data

(1) Stage (ft)	(2) Q (ft^3/s)	(3) S (Ac-Ft)	(4) $S_1-(O/2)\Delta t$ (Ac-Ft)	(5) $S_1+(O/2)\Delta t$ (Ac-Ft)
0.00	0	0	0.0	0.0
0.9	10	0.26	0.22	0.30
1.4	20	0.42	0.33	0.50
1.8	30	0.56	0.44	0.68
2.2	40	0.69	0.52	0.85
2.5	50	0.81	0.60	1.02
2.9	60	0.93	0.68	1.18
3.2	70	1.05	0.76	1.34
3.5	80	1.17	0.84	1.50
3.7	90	1.28	0.92	1.66
4.0	100	1.40	0.99	1.81
4.5	120	1.63	1.14	2.13
4.8	130	1.75	1.21	2.29
5.0	140	1.87	1.29.	2.44
5.3	150	1.98	1.36	2.60
5.5	160	2.10	1.44	2.76
5.7	170	2.22	1.52	2.92
6.0	180	2.34	1.60	3.08
6.4	200	2.58	1.76	3.41
6.8	220	2.83	1.92	3.74
7.0	230	2.95	2.00	3.90
7.4	250	3.21	2.17	4.24

Note: Δt = 10 min = 0.167 h, and 1cfs=0.0826 ac-ft/hr.

15.9 Example Problem (continued)

15.9.3 Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility are presented above in Table 15-6 that should provide adequate peak flow attenuation for runoff from both the 2- and 10-year design storms, which are shown in Tables 15-7 and 15-8. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2- and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Discharge values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 ft and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Storage routing was conducted for runoff from both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data given on the previous page and the Storage Characteristics Curves given on Figures 15-9 and 15-10, and 0.1-hour time steps are shown below for runoff from the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Table 15-7 Storage Routing For The 2-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ H_1 (Ac-Ft)	(4) $S_1-(O_1/2)\Delta t$ (ft)	(5)=(6-3) (Ac-Ft)	(6)=(3+5) $S_2+(O_2/2)\Delta t$ (Ac-Ft)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0.	0.0	0.00	0.00	0.00	0.00	0.00
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.08	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.421	0.47	1.03	18
1.1	2	0.02	1.30	0.327	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.0	0.70	0.15	0.15	0.40	3

15.9 Example Problem (continued)

Table 15-8 Storage Routing For The 10-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ (Ac-Ft)	(4) H_1 (ft)	(5) $S_1-(O_1/2)\Delta t$ (Ac-Ft)	(6) $S_2+(O_2/2)\Delta t$ (Ac-Ft)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0.00	0.0	0.00	0.0	0.0	0.00	0.00
0.1	50	0.21	0.21	0.0	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200 OK
0.5	90	1.05	5.80	1.30	2.35	4.95	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.0	0.90	0.22	0.22	0.60	6

For the routing calculations the following equation was used:

$$S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2\ \Delta t]$$

Also, column 6 = column 3 + column 5
And column 5=column 6 - column 3.

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 100-year storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements.

In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water and provisions for public safety.

15.9 Example Problem (continued)

15.9.4 Downstream Effects

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-year design storms.

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20%. As shown in Figure 15-12, the example results are well below 20%; downstream effects can thus be considered negligible and downstream flood routing omitted. However, it is important to be aware that the increased total volumes of water being released slowly over a longer period of time may contribute to bed and bank decay in the receiving channel.

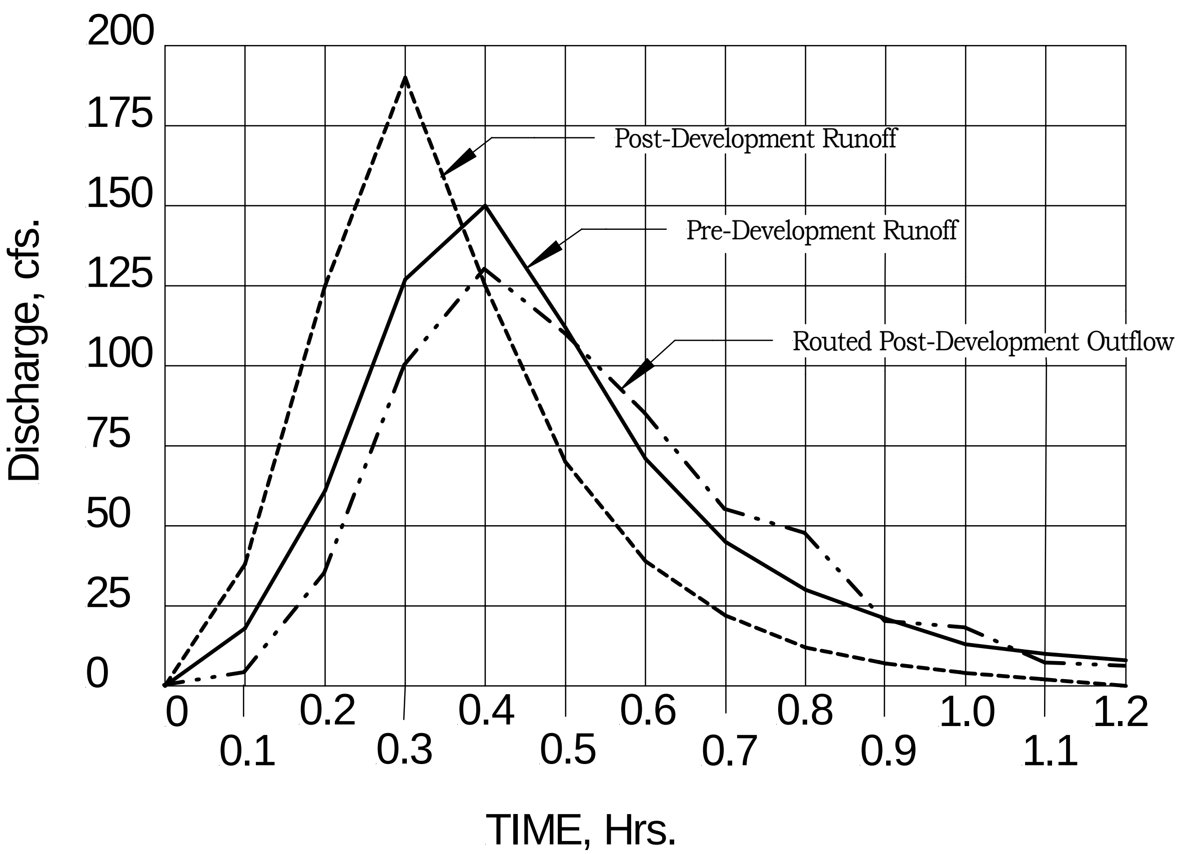

Figure 15-12 Runoff Hydrographs

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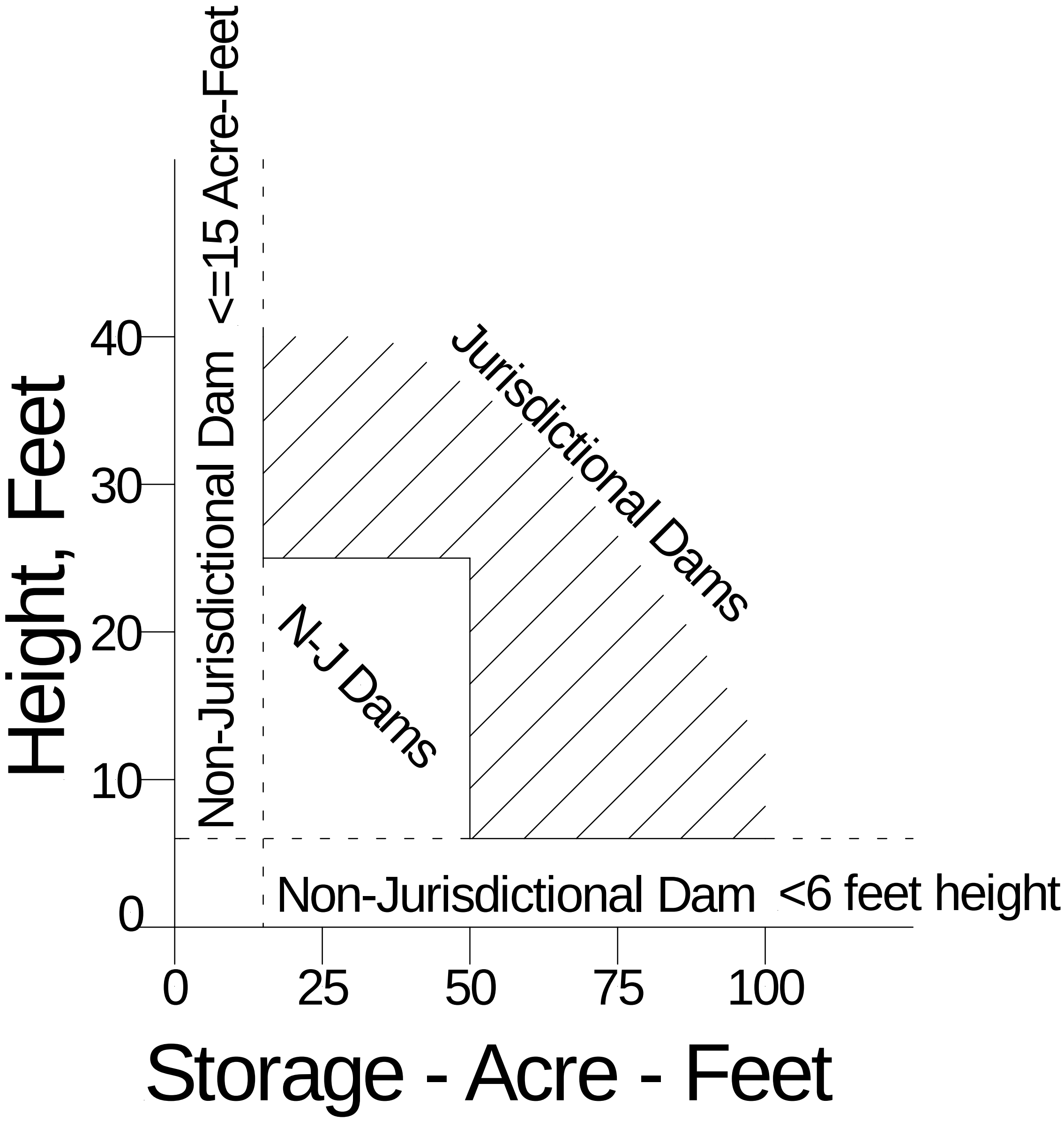
Appendix A ADWR Jurisdictional Dam Criteria



Appendix B HEC-1 Hydrograph for Rational Basin

There are times one will need a full 24 hour hydrograph for a basin that is analyzed using the Rational Method. The suggested procedure is described below. The procedure is a two-step process of first matching the volume of runoff and then matching the peak discharge. The procedure is demonstrated using the data from the Youngtown Basin – ADOT Hydrology Manual Page 3-19.

Youngtown Basin
Area = 0.13 sq. mi. = 83.2 Acres
Length = 4420 feet = 0.84 mi.
Urban watershed, K_b = 0.025 with ¼ acre lots
For 100 year rainfall, P1 = 2.53 in. C = 0.73

Assume slope = 1% = 52.8 ft/mi
$T_c = 11.4\ L^{0.5}K_b^{0.52}/S^{0.31} *i^{0.31}$
$T_c = 11.4\ (.84)^{0.5}(0.025)^{0.52}/(52.8)^{0.31} *i^{0.38}$
$T_c = 0.448/i^{0.38}$

Solving for T_c = 0.216 hr = 13 min., I = 6.8 in/hr.

Q = CiA = 0.73*6.8*83.2 = 413 cfs

Volume = $CP_{24}A$
Volume = $CP_{24}A$= 0.73*(3.8/12) *83.2 = 19.2 Ac-Ft

For HEC-1 with Tc = 0.216 hr,
$R = 0.37* T_c^{1.11}*L^{0.8}/A^{0.57}$
$R = 0.37* (0.216)^{1.11}*(0.84)^{0.8}/(0.13)^{0.57}$
R = 0.37*0.183*0.87/0.313 = 0.19

CP_{24}= 0.73*(3.8)=2.77
HEC-1 results:
Try 70% impervious
Runoff = 2.73 in, Qp = 280 cfs

Try 75% impervious
Runoff = 2.91 in, Qp = 296 cfs

Try 71% impervious
Runoff = 2.77 in, Qp = 283 cfs

Use 71% impervious and adjust T_c and R
Try T_c = 0.15 and R=0.14: Qp = 326 cfs
Try T_c = 0.10 and R=0.09: Qp = 390 cfs
Try T_c = 0.07 and R=0.06: Qp = 414 cfs, Use for hydrograph.